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HILL CITY - LEAD

SD PFH 17-1(6)

FINAL GEOTECHNICAL DESIGN REPORT

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Geotechnical Services Branch
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Federal Lands Highway
Commitment to Excellence

Technical Services

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

CENTRAL FEDERAL LANDS HIGHWAY DIVISION
LAKEWOOD, COLORADO

SIGNATURE SHEET

Report prepared by: ______________________________

Tracy D. Piparato, Highway Engineer

and: ______________________________

Marilyn Dodson, P.E., Geotechnical Engineer

Report reviewed by: ______________________________

Justin T. Henwood, P.E., Geotechnical Engineer

Approved for distribution by: ______________________________

Matthew J. DeMarco, Division Geotechnical Engineer

8-17-09
____

Date

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SECTION ONE
INTRODUCTION

1.1 PROJECT DESCRIPTION

The Central Federal Lands Highway Division (CFLHD) of the Federal Highway Administration (FHWA), in cooperation with the U.S. Forest Service (USFS), is proposing spot safety improvements spanning approximately 2.0 miles of South Dakota Forest Highway (FH) 17, known as Deerfield Road. This route is also known as Pennington County Road (CR) 308 and connects Hill City and Lead, located in the Black Hills National Forest, Pennington County, South Dakota.

The Hill City to Lead route project includes improvements at four locations along the first 3.5 miles of FH-17 north of its junction with US-385 in Hill City. These four locations are designated Sites 1 through 4 based on safety priorities, not project stationing. Currently, Site 1 and Site 2 are being programmed for FY2010 as project SD FH 17-1(6). Sites 3 and 4 are not included in the first project phase. Despite various levels of design efforts for each site, the geotechnical issues related to all four sites are addressed in this report. Location maps for the SD FH 17-1(6) Hill City – Lead project are included in Appendix A.

Site 1 begins at Station 31+00 and extends northerly to Station 49+00. This site was identified as the highest priority for safety improvements due to an undersized single-span bridge, an unsafe pedestrian crossing with the Mickelson Trail, and insufficient sight distance due to the horizontal alignment. The current roadway width is approximately 22 feet, and is planned to be widened to 28 feet. The paved roadway through this site is generally in fair condition with no significant cracking or distress. The existing single-span, wood-pile-supported bridge structure is located at approximately Station 38+65 over Newton Fork Creek. The replacement double barrel 10-feet by 10-feet reinforced concrete box culvert structure will be about twice as long as the existing structure, with a length of about 60-feet. The roadway alignment through this site consists of an at grade crossing of the Mickelson Trail at Station 40+32, two horizontal curves located just north and south of the trail crossing, and a large rock outcrop along the east side of FH-17. To improve the Mickelson Trail crossing, a 14-feet by 10-feet reinforced concrete box culvert approximately 115-feet long will be installed as an underpass. The slope ratios above the roadway range between 1V:1H in weathered schist bedrock and 1V:1.5H to 1V:2H in moderately vegetated soil slopes. The existing cut and fill slopes through this site appear stable.

Site 2 begins at Station 100+00 and extends northerly to Station 120+50. The roadway alignment through this site consists of an at grade crossing of the Mickelson Trail, two horizontal curves located just north and south of the trail crossing, an intersection with Burnt Fork Road with poor visibility, as well as a large rock outcrop along the east side of FH-17. The main safety improvements at this location will address a safer pedestrian crossing with the Mickelson Trail by constructing a 14-feet by 10-feet reinforced concrete box culvert underpass approximately 105-

feet long and a perpendicular intersection with Burnt Fork Road to improve visibility. The paved roadway through this site is generally in fair condition with no significant cracking or distress. The slope ratios above the roadway range between 2V:1H in weathered schist bedrock and 1V:1.5H to 1V:2H in moderately vegetated soil slopes. The existing cut and fill slopes through this site appear stable. Near Station 112+50, a short headwall was constructed to prevent erosion around a natural seepage area. Due to the raise in grade at this location to support the construction of the Mickelson Trail underpass, this area will be covered with about 25-feet of fill material. A special drainage plan was developed for this roadway section. A large rock cut exists from Station 105+70 to 107+00, which will be cut back about 15 feet, resulting in an approximately 50-foot- high cut. There is low rockfall risk anticipated at this location due to the maintenance history indicating good ditch catchment coupled with small-sized rock fragments eroding from the outcrop.

Site 3 begins at Station 49+00 and extends northerly to Station 64+81. The current roadway width is narrow, approximately 22 feet wide. The roadway alignment through this site is supported by a dry-stacked rock retaining wall located along Newton Fork Creek and contains short rock cuts adjacent to the inboard lanes with a narrow ditch. Settlement of the embankment backfill above the dry-stacked rock retaining wall has been occurring over several years, with estimates of about 3-feet of total vertical displacement that is patched annually. Movement within the retaining wall backfill is also causing deformation of the guardrail at the shoulder of the roadway. Due to the proximity of Newton Fork Creek at the base of the retaining wall, undermining of the foundation from stream scour may also be a contributing factor to the movement expressed in the roadway. Replacement of the existing retaining wall with a mechanically stabilized earth (MSE) wall is recommended. The paved roadway through this site is generally in fair condition with significant cracking and distress only located above the dry-stacked retaining wall from Station 57+80 to 61+55.

Site 4 begins at Station 120+50 and extends northerly to Station 137+00. The roadway alignment through this site consists of a dry-stacked rock retaining wall, a steep grade hill, a horizontal curve following the grade change, and a pond and dam structure with a pull-off and parking lot at the bottom of the grade change. The paved roadway through this section is in fair condition with no significant cracking or distress with the exception of Station 131+50 to 132+80, immediately above the retaining wall and just downstream of the outlet from Newton Pond. Pennington County plans on draining the pond and restoring the natural stream channel, but that has not occurred to date. The existing roadway embankment is also serving as a retaining structure for the east side of Newton Pond, so roadway improvements in this area are not to disturb the existing embankment due to the potential development of issues beyond the scope of this roadway improvement project. A rockery wall would support the widening of the roadway into the Mickelson Trail embankment slope near Newton Pond so no impact is made to the dam area. Additionally, the existing dry-stacked retaining wall would be replaced with a short MSE wall to alleviate settlement problems.

1.2 BACKGROUND DOCUMENTS

No previous subsurface investigations have been completed by CFLHD. However, an initial project scoping visit was made on July 25, 2007, to review the current conditions and rehabilitation options for the project route. In the Scoping Trip Report from this site visit, the following key roadway improvement observations were made:

1. The existing paved roadway width is substandard (22 ft wide) and will require widening to achieve the planned 28-foot paved width.

2. Horizontal and vertical re-alignment, that will include cuts and fills, will be required in certain sections to increase the speed limit, improve drainage, and improve the safety of the Mickelson Trail crossings. Several cuts will be in exposed bedrock material.

3. Two existing dry-stacked rock retaining walls along Newton Fork Creek are failing and will need to be replaced to support the roadway embankment.

4. Settlement problems exist in the roadway above both of the existing dry-stacked rock retaining walls.

A Final Hydraulics Report by the Hydraulics Section of CFLHD was published in March, 2008, with the results of the hydrologic and final hydraulic analysis performed to identify culverts, outlet protection, and bridge scour for the SD PFH 17-1(6) project. A supplemental memo detailing culvert recommendations and additional bridge scour analysis was published in February, 2009.

Pavement recommendations were developed based on the subsurface investigation presented in this report. Since most of the drilling along the route was targeted for geotechnical issues, a few pavement boreholes were added to this investigation to save both time and money by combining resources and only mobilizing the drill rig to the site one time. The Pavements Recommendations Memo was published on December 8, 2008.

Information contained in these documents was used to support the preparation of this report.

1.3 PURPOSE

This investigation was conducted to complete the following tasks:

1. Characterize surface and subsurface soil and rock conditions for the route.
2. Identify shrink/swell factors, cut slope ratios, and fill slope ratios for the route.
3. Identify rock cut slope areas and provide recommendations for their construction.

4. Evaluate settlement areas associated with two existing dry-stacked retaining walls (Sites 3 and 4) and provide mitigation recommendations.
5. Provide culvert foundation and associated wing wall retaining structure recommendations for three concrete box culverts: one placed at Newton Fork Creek along FH 17, and two for the Mickelson Trail crossing FH-17.
6. Evaluate the suitability of the site material for constructing cut and fills.

SECTION TWO
GEOLOGY

2.1 REGIONAL AND LOCAL GEOLOGY

South Dakota Forest Highway 17, known as Deerfield Road, lies on the western side of the Missouri River which divides the state from north to south. The western portion of the state of South Dakota is characterized by rough terrain, thin soil, and sparse rainfall. Deerfield Road is located near the geographical center of the Black Hills.

The Black Hills are a small range of mountains, but extend from the Great Plains of North America in western South Dakota south into Wyoming. The Black Hills are set off from the main body of the Rocky Mountains and are home to the tallest peaks of Continental North America east of the Rockies. The rock formations of the Black Hills began as various layers of limestone deep within the earth. These formations form concentric rings around the center which is composed mostly of Precambrian Granite and metamorphic rocks. The Mount Rushmore National Memorial, located less than 20 miles from Hill City, is carved out of the Harney Peak granite batholith. The Crazy Horse Memorial is being carved out of pegmatitic granite of the same formation. The first ring surrounding this core of granite is comprised mainly of metamorphic rocks. These metamorphic rocks began as muddy sandstones and now appear as gray schists. Next are the Paleozoic rocks of the Deadwood Formation, consisting of sandstone, limestone, and green shales. These units are generally located west of the project site. The Paleozoic rocks are then followed by the Ordovician rocks which include the Winnepeg green shale and buff-colored dolomite of the Whitewood Formation. A regional geologic map of the project vicinity is presented in Appendix A.

The project location is mainly within the metamorphosed sedimentary rocks, mostly comprised of weathered schists. Subgrade soils encountered along the project generally consisted of silt, silty sand, and sandy, silty clays.

2.2 GEOLOGIC HAZARDS

Most of the project area is underlain by Precambrian Metamorphic schist with a generally thin layer of silty sand or silty clay. Schist decomposes to form silty sand and clays. Potential geologic hazards that could be associated with roadway construction in these units include swelling soils associated with high clay content, frost-heave associated with silty fine sands, weak subgrade soils due to both fine-grained soils and water, rockfall, and slope instability at cut and fill slope locations.

Seismic activity in the Hill City-Lead area is considered to be low (Greis, 1996). Additionally, there are no identified active faults in the project area (USGS, 2007). Recommended seismic response parameters for use in design are based on the American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications, 4th edition, 2008

Interim and represents horizontal peak ground acceleration (PGA) with 7 percent probability of exceedance in 75 years (approximately 1,000-year return period). A site location of 43.917ºN latitude and -103.627ºW longitude was used to obtain a PGA of 0.021. The site soils are classified as **Class D** according to the site class definitions specified in table 3.10.3.1-1 of ASSHTO. Although shear wave velocities of the soils were not measured, the top 100 feet of material consists of bedrock overlain with silty sand and sandy silt. A Site Factor of 1.6 was applied to the PGA, resulting in an acceleration coefficient of 0.034. This value corresponds to a **Seismic Zone 1** according to Table 3.10.6-1. AASHTO recommends that structures with acceleration coefficients less than or equal to 0.05 (in Seismic Zone 1) need not be analyzed for seismic loads, regardless of their importance and geometry (Section 4.7.4).

3.1 PROCEDURES

The following sections describe the procedures used to complete the exploratory drilling, sampling, rock cut mapping, and laboratory testing as part of the Hill City – Lead project.

3.1.1 Exploratory Drilling

Geotechnical boreholes, labeled B-1 through B-30, were drilled along the route to characterize subsurface conditions and obtain samples for laboratory testing. Five boreholes (B-7, B-8, B-10, B-11, and B-12) in this sequence were originally planned, but not drilled due to limited site access. One borehole, B-14A, was added during drilling to collect more information above the existing retaining wall at Site 3. Geotechnical borehole depths ranged from approximately 1.5 feet to 31.5 feet, while pavement subgrade investigation boreholes were advanced approximately 5 feet deep. Table 3-1 summarizes the location and purpose of each borehole. Borehole location plans are included in Appendix B.

American Technical Services of Black Hawk, South Dakota, conducted the drilling using two CME 55 truck-mounted drill rigs. Borings were located and logged by CFLHD personnel. All of the borings were advanced using hollow stem augers through the on-site soils to bedrock contact and auger refusal. Bedrock was not drilled or sampled as part of this site investigation. Geophysical surveys of the bedrock cut areas were planned as part of the subsurface investigation, but equipment hardware malfunction prevented the collection of any data. A second mobilization of track-mounted drill rigs for bedrock subsurface investigation was not carried out due to project budget and time constraints, as well as relatively low risk of unexpected bedrock conditions moving forward to construction with the information collected at the surface outcrops of the bedrock material. Assumptions for the preparation of the project plans would unlikely change with the addition of more bedrock condition assessment.

Sampling of materials beneath the tip of augers was performed as borings were advanced. Sampling was typically conducted at 5-foot intervals to the termination depth of the boring or auger refusal. Soil samples were recovered with a 2-inch outside diameter split-barrel sampler or modified California liners in accordance with AASHTO T 200. Representative portions of recovered samples were preserved for laboratory testing. The sampling sequence for the borings is summarized on the boring logs attached in Appendix C. A graphic summary log for each of the four sites is also included at the front of Appendix C to illustrate the variability of the materials with depth.

Standard penetration tests (SPT) were performed and resistances were recorded during the recovery of each split barrel sample, in accordance with AASHTO T 206. The sampler was driven into the soil using an automatic hammer. Sample recovery measurements were made and recorded for each sampling attempt. A field description by color and texture was made for each recovered sample.

The results of field tests and measurements were recorded on field logs and appropriate data sheets at the time of the investigation. The data sheets and logs contain information concerning the boring methods; samples attempted and recovered; indications of the presence of various materials (i.e., clay, sand, gravel, boulders) and observations of groundwater. They also contain interpretations by the field personnel of the conditions based on the performance of the drilling equipment and cuttings brought to the surface. Therefore, the field data represent both factual and interpretative information.

The boring logs represent a compilation of field and laboratory data and description of the soil samples. These records occasionally do not include all data recorded on field logs and data sheets, but do include all information considered relevant to the design and construction recommendations, as contained in this report. Borehole logs and photographs of the geotechnical borehole locations are provided in Appendices C and E, respectively.

Table 3-1. Borehole Summary

Borehole No.	Station	Offset (ft)	Explored Depth (ft)	Project Site	Purpose
B-1	30+83	6 RT	9.0	Site 1	Pavements and soil classification
B-2	38+45	12RT	27.0	Site 1	Box culvert foundation material (bridge replacement)
B-3	40+47	4 RT	30.0	Site 1	New Box culvert foundation material (trail crossing)
B-4	41+28	6 RT	13.0	Site 1	Embankment fill area
B-5	44+96	14.5 RT	7.5	Site 1	Cutslope/ soil-bedrock contact depth
B-6	47+17	26.5 RT	1.5	Site 1	Cutslope/ soil-bedrock contact depth
B-9*	Access Rd @ 12400	260.0 RT	5.0	Site 1	Cutslope/ soil-bedrock contact depth
B-13	51+93	6 RT	11.5	Site 3	Pavement/ embankment properties
B-14	57+70	7 LT	14.2	Site 3	MSE wall/ classification/ bedrock contact
B-14A*	59+20	7.5 LT	10.5	Site 3	MSE wall/ classification/ bedrock contact
B-15	59+70	10.5 LT	17.5	Site 3	MSE wall/ classification/ bedrock contact
B-16	60+27	2 RT	12.0	Site 3	MSE wall/ classification/ bedrock contact
B-17	60+68	10.5 LT	21.3	Site 3	MSE wall/ classification/ bedrock contact
B-18	61+70	10.0 LT	20.6	Site 3	MSE wall/ classification/ bedrock contact
B-19	102+20	55.5 LT	11.5	Site 2	Bike path/ classification
B-20	104+40	6.5 RT	11.5	Site 2	Pavement/ classification
B-21	110+00	17.5 LT	10.0	Site 2	Pavement/classification
B-22	111+00	14.5 RT	12.5	Site 2	New Box culvert foundation material (trail crossing)
B-23	112+25	11.5 RT	28.5	Site 2	Bike path/ classification
B-24	Burnt Fork Rd/ Sta. 81+50	Centerline	6.5	Site 2	Base of large embankment fill/ classification
B-25	117+00	Centerline	10.8	Site 2	Pavement/ classification
B-26	124+00	4 RT	11.5	Site 4	Pavement/ classification
B-27	131+00	4 RT	14.0	Site 4	Base of rockery/ foundation, material classification
B-28	132+15	8 LT	15.5	Site 4	MSE wall/ classification
B-29	133+00	6.5 RT	20.6	Site 4	Base of rockery/ foundation, classification
B-30	Mickelson Trail Near 131+40	Centerline of Trail	31.5	Site 4	Above roadway along bike path/ behind rockery/ material classification

*Note: B-7, B-8, B-10, B-11, B-12 were not drilled due to limited site access with truck-mounted drill rigs. B-14A was added during drilling.

3.1.2 Rock Cut Mapping

A geologic reconnaissance was conducted to evaluate geologic conditions and rock structure in the areas where the roadway re-alignment may require cut slopes in rock. Concurrent with the subsurface investigation, exposed bedrock cuts were mapped at the surface at Sites 1, 2, and 3. Rock structural mapping entailed observing and measuring engineering characteristics and orientation of rock discontinuities that make up the rock mass. A “window” mapping technique was used to record the general orientation and variability of each discontinuity set at each rock cut location, not a detailed scanline survey. Additionally, existing ditch widths were measured and observations were made regarding their ability to catch material weathering from the cut slopes. Local maintenance crews have no records of rock falls that have extended beyond the existing ditches, which only require infrequent cleaning. Photographs of each rock cut are included in Appendix G. Discussion of the kinematic (stereonet) analysis, stability analysis, and recommendations for each rock cut are included in the Analysis and Recommendations Section of this report, Section 4.2.

3.1.3 Laboratory Testing

At the conclusion of the fieldwork, index tests were conducted on soil samples recovered from completed borings. Laboratory testing was performed by the CFLHD Materials Laboratory. Tests on the samples included gradation (AASHTO T-27), Atterberg limits (AASHTO T-89, T-90), hydrometer, direct shear (ASTM D3080), pH (AASHTO T-289), resistivity (AASHTO T-288), sodium sulfate soundness (AASHTO T-104), and moisture-density (AASHTO T-99). Results of these tests were used to classify the soils according to AASHTO M-145 and ASTM D 2487 (Unified Soil Classification System) to verify field logs, which were then updated as required. Classification in this manner provides an indication of the soil’s mechanical properties. Laboratory test results are summarized in Appendix D.

3.2 RESULTS

The following sections present the results of the exploratory drilling, laboratory testing, and site reconnaissance that were conducted for this project. Generally, the materials across the four sites on the project were fairly consistent, comprising three distinct layers: roadway fill, low-plasticity silty sand alluvium, and weathered schist bedrock. The overburden material at Site 4 generally contains more clay than the other three sites, but still classifies as clayey silt with moderate plasticity. This type of silty sand material is typically susceptible to frost heave and piping issues.

3.2.1 Laboratory Test Results

Laboratory test results for soil samples that were collected during drilling are summarized in Table 3-2 through Table 3-7. Detailed test results, including gradation curves, are presented in Appendix D.

Table 3-2. Physical Property Data

Borehole	Location			Dry Density (pcf)	Moisture Content (%)	Class. (AASHTO) (USCS)	Gradation (%)			Atterberg Limits		R-Value
	Station	Offset (ft)	Depth (ft)				Gravel >#4	Sand <#4 >#200	Silt & Clay <#200	LL (%)	PI (%)	
B-1 Site 1	30+83	6 Right	1.2-5.0	---	---	A-4 ML	7	26	67	NV	NP	---
B-2 Site 1	38+45	12 Right	10.0-11.5& 15.0-16.5	---	---	A-4 SM(1)	23	35	42	28	8	---
B-4 Site 1	41+28	6 Right	1.0-5.0	---	---	A-4 CL-ML	7	33	60	24	7	29
B-9 Site 1	Access Rd @12400	260 Right	0.0-5.0	---	---	A-4 ML	10	28	62	NV	NP	---
B-13 Site 3	51+93	6 Right	0.0-5.0	---	---	A-4 CL-ML	12	31	57	25	6	---
B-14 Site 3	57+70	7 Left	11.5-15.0	---	---	A-2-4 SC-SM	31	34	35	23	4	---
B-16 Site 3	60+27	2 Right	0.0-5.0	131	8	A-2-4 SC-SM	21	44	35	24	5	19
B-19 Site 2	102+20	55.5 Left	5.0-6.5	---	---	A-4 CL	4	13	83	28	9	---
B-21 Site 2	110+00	17.5 Left	0.0-5.0	---	---	A-4 SC/CL(2)	18	37	49	27	8	23
B-22 Site 2	111+00	14.5 Right	5.0-10.0	---	---	A-1-b SM	28	48	24	22	3	---
B-23 Site 2	112+25	11.5 Right	25.0-26.5	---	---	---	9	58	33	---	---	---
B-25 Site 2	117+00	Centerline	0.0-5.0	---	---	A-2-4 SC-SM	31	36	33	24	6	23
B-29 Site 4	133+00	6.5 Right	10.0-11.5	---	---	A-6 ML(3)	3	17	80	32	14	---
B-30 Site 4	131+40 Mainline Station	Mickelson Trail Centerline	5.0-6.5 & 10.0-11.5& 15.0-16.5	---	---	A-6 CL	2	22	76	30	12	---

Notes: NV = No value; NP = Non-plastic; -- = Not applicable/ no test was conducted
(1)= Classification changed from SC to SM based on results from hydrometer test
(2)= Classification changed from SC to SC/CL based on percent fines within 2% of classification boundary
(3)= Classification changed from CL to ML based on results from hydrometer test

Two geochemical tests were performed on representative material samples to evaluate their potential to corrode buried steel structures and concrete. Testing for resistivity and pH were performed in general accordance with AASHTO T 288 and T 289, respectively. A summary of the test results is provided in Table 3-3 and detailed test results are presented in Appendix D.

For structural and drainage elements, a soil is considered to be "mildly corrosive" if the resistivity is greater than 5,000 ohm-cm, as defined by FHWA-NHI-00-044 (Elias, 2000). Tests for sulfate and chloride content are not required when pH is between 6.0 and 8.0 and resistivity if greater than 5,000 ohm-centimeters. Soils with a resistivity greater than 5,000 ohm-cm indicates that the sulfate and chloride contents are low, resulting in a low corrosion potential. In general, the tested soils from the project sites exhibited negligible and mildly corrosive aggressiveness towards concrete structures and buried steel, respectively.

The potential corrosive environment for metal placed beneath the ground surface at the site was evaluated based on the data collected during the field exploration and criteria presented in the Federal Lands Highways "*Project Development and Design Manual*" (FLH, 2008) for culverts and FHWA-NHI-00-044 (Elias, 2000) for MSE wall backfill material.

For culverts, the electrical resistivity measurements indicate values above 1,500 ohm-centimeters and pH tests indicate values between 5.0 and 9.0. These test results indicate no corrosive restrictions will be necessary for the type of pipe culverts used on the project.

For MSE structures with metallic reinforcements, select granular backfill material is required to meet the following corrosion related requirements according to FHWA Standard Specifications FP-03, which is based on recommendations from FHWA-NHI-00-044 (Elias, 2000):

- Resistivity, AASHTO T 288 3,000 ohm-cm minimum
- pH, AASHTO T 289 5.0 to 10.0
- Sulfate content, AASHTO T 290 200 ppm maximum
- Chloride content, AASHTO T 291 100 ppm maximum

MSE backfill material that meets the above criteria will be considered non-corrosive to both the metallic soil reinforcement as well as the reinforced concrete retaining wall. Electrochemical testing indicates the on-site material will meet these corrosion requirements for select granular backfill material, but the physical properties will not. Therefore, it should be anticipated that MSE backfill material will need to be imported for work at Sites 3 and 4.

A low concentration of water soluble sulfates represents a negligible degree of sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of negligible, moderate, severe, and very severe as presented in the American Concrete Institute (ACI, 2005) *Building Code Requirements for Structural Concrete*. Based on this information, special sulfate resistant cement will not be required for concrete exposed to the on-site soils. Type II cement is recommended for the precast box culverts, headwalls, and wingwalls.

Table 3-3. Electrochemical Property Data

Borehole	Location			Class. (AASHTO) (USCS)	pH	Resistivity (ohm-cm)	Sulfates* (ppm)	Chlorides* (ppm)
	Station	Offset (ft)	Depth (ft)					
B-2	60+27	2 Right	0.0-5.0	A-4 SM	7.9	6050	---	---
B-22	111+00	14.5 Right	5.0+10.0	A-1-b SM	8.3	5370	---	---

Note: * Tests for sulfate and chloride content are not required when pH is between 6.0 and 8.0 and resistivity is greater than 5,000 ohm centimeters.

The Direct Shear test (ASTM D3080) was conducted on one sample from Site 3, borehole B-16 collected at a depth of 5.0 to 10.0 feet (Table 3-4). This sample was selected because it displayed typical physical properties for the material encountered along the proposed MSE wall location. Surcharge values of 7 psi, 14 psi, and 28 psi were used to test the soil sample, which correspond to a maximum anticipated MSE wall height of 16 feet and a soil unit weight of 125 pcf. Select Wall Backfill Material for MSE walls requires a peak friction angle of 34-degrees. This test result indicates that the on-site material will not meet the requirements for Select Wall Backfill Material for the MSE walls.

Table 3-4. Direct Shear Test Results

Borehole	Location			Class. (AASHTO) (USCS)	Peak		Ultimate	
	Station	Offset (ft)	Depth (ft)		Cohesion (psf)	Phi (degrees)	Cohesion (psf)	Phi (degrees)
B-16	38+45	12 Right	5.0-10.0	A-2-4 SC-SM	288.6	33.4	54.6	35.4

The Sodium Sulfate Soundness test (AASHTO T-104) was conducted on one sample from borehole B-5 collected at a depth of 1.2 to 5.0 feet (Table 3-5). Test results indicate 1% loss during the test. A corresponding Los Angeles Abrasion test (AASHTO T-96) was not conducted due to the small amount of material greater than the #4 sieve (about 10%), which is required to conduct the test.

Table 3-5. Durability Test Results

Borehole	Location			Class. (AASHTO) (USCS)	Sodium Sulfate Soundness (% loss)	Los Angeles Abrasion*
	Station	Offset (ft)	Depth (ft)			
B-5	44+96	14.5 Right	1.2-5.0	A-4 SM	1	---

Note: * The Los Angeles Abrasion test was not conducted due to the small amount of material greater than the #4 Sieve required to conduct the test.

A compaction test (AASHTO T-99, Method C) was conducted on one sample from borehole B-16 collected at a depth of 0.0-5.0 feet (Table 3-6). The maximum dry density was determined to be 131 pcf with a corresponding optimum moisture content of 8 percent. This sample classified as silty, low-plasticity clayey, course to fine sand with some fine gravel (A-2-4 by AASHTO and SC-SM by USCS), which is fairly typical of materials encountered across all of the sites on the project.

Table 3-6. Compaction Test Results

Borehole	Location			Class. (AASHTO) (USCS)	Maximum Dry Density (psf)	Optimum Moisture Content (%)
	Station	Offset (ft)	Depth (ft)			
B-16	60+27	2 Right	0.0-5.0	A-2-4 SC-SM	131	8.0

Note: * AASHTO T-99, Method C.

Hydrometer tests were performed on two samples, one from borehole B-2 at a depth of 10.0 to 16.5 feet, and the other from B-29 at a depth of 10.0-11.5 feet (Table 3-7). For the B-2 sample, 42% of the sample passed the #200 sieve with 17% clay and 25% silt. As indicated in the note on Table 3-2, the classification for this sample was changed from clayey sand to silty sand based on the higher percent of silt in the total percent fines. For the B-29 sample, 80% of the sample passed the #200 sieve with 31% clay and 49% silt. Also indicated in the note on Table 3-2, the classification for this sample was changed from medium plasticity clay to clayey silt based on the higher percent of silt in the total percent fines. Gradation curves illustrating this data are presented in Appendix D.

Table 3-7. Hydrometer Test Results

Borehole	Location			Class. (AASHTO) (USCS)	Sample <#200 (%)	Silt (%)	Clay (%)
	Station	Offset (ft)	Depth (ft)				
B-2	38+45	12 Right	10.0-16.5	A-4 SM(1)	42	25	17
B-29	133+00	6.5 Right	10.0-11.5	A-6 ML(2)	80	49	31

Notes: (1) = Classification changed from SC to SM based on results from hydrometer test
(2) = Classification changed from CL to ML based on results from hydrometer test

3.2.2 Site 1 – Station 31+00 to Station 49+00

A total of seven boreholes, B-1 through B-6 and B-9, were completed along Site 1, as illustrated in the Borehole Location plan in Appendix B and the Graphic Summary Logs in Appendix C. The boreholes generally encountered brown silt to silty sand roadway fill material to depths ranging from 4.0 to 10.5 feet below the ground surface. Uncorrected SPT N-values conducted in the fill material ranged from 1 to 3, with the material generally being soft or loose. The R-value for the silty clay (A-4) at this site was 29.

Alluvium, consisting of low-plasticity silts, silty sands, and silty clays, was encountered beneath the roadway fill to depths ranging from 4.0 to 24.0 feet below the ground surface. Uncorrected

SPT N-values conducted in the fill material ranged from 12 to 50, with the material generally being medium dense.

Bedrock, consisting of gray, moderately to highly weathered schist, was encountered beneath the roadway fill or the alluvium to depths ranging from 4.0 to 30.0 feet, the borehole termination depths. Decomposed schist generally grades to harder rock with depth, which was reflected in the drilling recovery that was usually 100 percent until auger refusal.

Borehole B-2 was drilled near the south abutment of the existing bridge that will be replaced with a double barrel 10-feet by 10-feet reinforced concrete box culvert. At this location, silty sand material was encountered to a depth of about 25 feet below the ground surface, then gray, weathered schist was encountered until auger refusal at 27.0 feet below the ground surface. The anticipated bearing elevation of the culvert corresponds to an approximate depth of 15 feet in the borehole, which is about 10 feet above the bedrock contact.

Borehole B-3 was drilled near the Mickelson Trail crossing where a new 10-feet by 14-feet reinforced concrete box culvert underpass will be constructed. At this location, silty sand material was encountered to a depth of about 21.5 feet, and then gray weathered schist was encountered until auger refusal at 30.0 feet below the ground surface. The anticipated bearing elevation of the culvert corresponds to an approximate depth of 10 feet in the borehole, which is about 11.5 feet above the bedrock contact.

Groundwater was encountered in two of the deeper boreholes that were the closest to the Newton Fork Creek. Groundwater was encountered in borehole B-2 at approximately 20 feet below the ground surface and in borehole B-3 at approximately 19 feet below the ground surface. These elevations visually appeared to correspond to the surface water elevation of the water in Newton Fork Creek at the time of drilling.

3.2.3 Site 2 – Station 100+00 to Station 120+50

A total of seven boreholes, B-19 through B-25, were completed along Site 2, as illustrated in the Borehole Location plan in Appendix B and the Graphic Summary Logs in Appendix C. The boreholes generally encountered brown silt to silty sand roadway fill material to depths ranging from 5.0 to 15.0 feet below the ground surface. Representative soil samples obtained from the borings classified as low-plasticity clays, clayey sands, and silty sands, which corresponds to A-4, A-1-b, and A-2-4 in the AASHTO classification system. The R-value for the clayey sand (A-4) and the clayey/silty sand (A-2-4) at this site were both 23. Uncorrected SPT N-values conducted in the fill material ranged from 5 to 15, with the material generally being medium dense.

Bedrock, consisting of gray, moderately to highly weathered schist, was encountered beneath the roadway fill or the alluvium to depths ranging from 5.0 to 25.0 feet below the ground surface, the borehole termination depths. In boreholes B-19, B-20, and B-25, bedrock was not encountered at the termination depths of about 11.5 feet below the ground surface.

Borehole B-22 was drilled near the Mickelson Trail crossing where a new 10-feet by 14-feet reinforced concrete box culvert underpass will be constructed. At this location, silty sand material was encountered to a depth of about 10.0 feet below the ground surface, then gray, weathered schist was encountered until auger refusal at a depth of 12.5 feet below the ground surface. The anticipated bearing elevation of the culvert corresponds to an approximate depth of 12 feet in the borehole, which is near the bedrock contact.

Groundwater was encountered in borehole B-23 at approximately 21.5 feet below the ground surface. This elevation visually appeared to correspond to the surface water elevation of the water in Newton Fork Creek at the time of drilling.

3.2.4 Site 3 – Station 49+00 to Station 100+00

A total of seven boreholes, B-13 through B-18, including borehole B-14A, were completed along Site 3, as illustrated in the Borehole Location plan in Appendix B and the Graphic Summary Logs in Appendix C. The boreholes generally encountered brown silt to silty sand roadway fill material to depths ranging from 10 to 15 feet below the ground surface. Representative soil samples obtained from the borings classified as low-plasticity silty clays, clayey sands, and silty sands, which corresponds to A-4 and A-2-4 in the AASHTO classification system. Occasional cobble and boulder layers were encountered with artificially high blow counts due to the sampler hitting on rock larger than its diameter, rather than driving through the soil section. The R-value for the silty/clayey sand (A-2-4) at this site was 19. Uncorrected SPT N-values conducted in the fill material ranged from 2 to 8, with the material generally being medium dense.

Bedrock, consisting of gray, moderately to highly weathered schist, was encountered beneath the roadway fill or the alluvium to depths ranging from 10.0 to 15.0 feet below the ground surface, the borehole termination depths. Decomposed schist generally grades to harder rock with depth, which was reflected in auger refusal within 2 feet of the bedrock surface.

Groundwater was encountered in boreholes B-17 and B-18, at approximately 10.5 feet below the ground surface. This elevation visually appeared to correspond to the surface water elevation of the water in Newton Fork Creek at the time of drilling.

3.2.5 Site 4 – Station 120+50 to Station 137+00

A total of five boreholes, B-26 through B-30, were completed along Site 4, as illustrated in the Borehole Location plan in Appendix B and the Graphic Summary Logs in Appendix C. The boreholes generally encountered brown silt to silty sand roadway fill material to depths ranging from 4.0 to 10.5 feet below the ground surface. Representative soil samples obtained from the borings classified as low-plasticity silty sands, clayey silts, and silty clays, which corresponds to A-6 in the AASHTO classification system. Uncorrected SPT N-values conducted in the fill material ranged from 2 to 15, with the material generally being loose to medium dense.

Bedrock, consisting of gray, moderately to highly weathered schist, was encountered beneath the roadway fill or the alluvium to depths ranging from 11.5 to 16.0 feet below the ground surface, the borehole termination depths.

Borehole B-30 was drilled on the Mickelson Trail, above the roadway, through fill material that will be retained by the proposed rockery wall. The borehole encountered brown silty clay to clayey silt with some fine to course sand and a trace of fine gravel from the ground surface to 31.5 feet below the ground surface, at the borehole termination depth.

Groundwater was encountered in boreholes B-28 and B-30, at approximately 7.5 feet and 25 feet below the ground surface, respectively. This elevation visually appeared to correspond to the surface water elevation of the water in the stream released from Newton Pond at the time of drilling.

4.1 CULVERT DESIGN RECOMMENDATIONS

Three precast, concrete box culverts are planned on the project. Site 1 has two of the culverts, with one planned as a bridge replacement at the Newton Fork Creek crossing at Station 38+36, and the other as an underpass structure at the Mickelson Trail crossing at Station 40+32. Site 2 also has a new Mickelson Trail underpass structure at Station 110+81, which will be identical in cross section to the Site 1 trail crossing. The wingwalls of the Site 2 underpass structure will have a simulated stone masonry veneer due to the high visibility location.

At Site 1, borehole B-2 was drilled near the planned bridge replacement location, which will be a double box 10-foot by 10-foot reinforced box culvert with a bearing elevation of approximately 5,013.0 feet above mean sea level. Borehole B-3 was drilled near the Mickelson Trail crossing, where a new 10-foot by 14-foot reinforced box culvert will be placed with a bearing elevation of approximately 5,017.0 feet above mean sea level. At Site 2, borehole B-21 was drilled near the Mickelson Trail crossing, where a new 10-foot by 14-foot reinforced box culvert will be placed with a bearing elevation of approximately 5,224.0 feet above mean sea level.

Subsurface information from the closest boreholes to the structure locations and laboratory test results suggest that the subsurface can be characterized for design purposes as silty sand. At Site 2, there is a chance of intersecting moderately weathered schist bedrock on the north side of the culvert, but the exact location of the bedrock will not be known until excavation begins during construction. Due to anticipated light loading conditions of the proposed culverts, shallow foundations are recommended for the foundations of all three culverts.

Index property and direct shear tests for the silty sand were analyzed and referenced with presumptive values found in many commonly used engineering manuals (AASHTO, NAVFAC, etc). Table 4-1 below contains the recommended design material properties. The direct shear test was conducted on a typical silty sand sample with 35-percent fines. Due to the variability of the percent fines across the site, a conservative design friction angle value of 29-degrees was selected, which is lower than the tested peak friction angle of approximately 33-degrees (Table 3-4).

Table 4-1. Design Material Properties

Soil Description	Location	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Retained Fill/ Silty Sand (SC/SM)	Site 1, 2, and 3	120	29	0
Sandy Clay/ Sandy Silt (CL/ML)	Site 4	110	29	50
Weathered Schist	Site 1, 2, 3, 4	130	23	1000
Reinforced Fill / Structure Backfill (MSE)	Site 3 and 4	125	34	0

Rockery[1]	Site 4	130	40	1000

Note: (1)Rock-on-rock values assumed

4.1.1 Bearing Resistance

The following design and construction details should be observed for spread footings placed on silty sand material:

1. Spread footing foundations for the proposed precast reinforced concrete box culvert structures can be designed for a presumptive bearing resistance of 3 ksf at the Service Limit State for Load Resistance Factor Design (LRFD). A total settlement of less than 1.0 inches was estimated if the above presumptive bearing pressure is used.
2. For the Strength Limit State, a nominal bearing pressure of 15 ksf and a resistance factor of 0.45 should be applied. Table 4-2 contains the recommended resistance factors for each limit state.

Table 4-2. Resistance Factors

	Resistance Factor[1], φ		
Limit State	**Bearing**	**Shear Resistance to Sliding**	**Passive Pressure Resistance to Sliding**
Strength	0.45[2]	0.80[3]	0.50
Service/ Extreme Event	1.00	1.00	1.00

Notes: (1) Resistance Factors from AASHTO LRFD Bridge Design Specifications, 2008 Interim, Tables 10.5.5.2.2-1
(2) Theoretical method, in sand, using SPT
(3) Cast-in-Place concrete on sand

3. The base of the footing should be located below the depth of frost potential, which is 2-feet based on local practices. The interior support footing bearing elevation should be a minimum of 3 feet below the scour elevation.
4. End support footings should be protected from erosion by construction of wing walls or equivalent alternative.
5. At the Site 1 Newton Fork Creek crossing, inlet and outlet cutoff walls are recommended to be a minimum of 2-feet deep to prevent scour.
6. At Site 2, weathered bedrock may be encountered at the footing elevation during construction. If this occurs, there should be a minimum thickness of 1-ft of compacted foundation fill below the bearing elevation of the culvert or wing wall footings to ensure uniform distribution of the foundation loading and prevent differential settlement due to variable bedrock surface elevations.

7. Dewatering and/or re-routing of the creek may be required to allow construction of the interior footing to proceed ‘in the dry”. Minor re-routing of the stream channel is anticipated due to summer construction and relatively low stream flow.

4.1.2 Friction Factor

For LRFD design, the friction factor to be used in conducting the sliding analysis of the foundation or footing is 0.69, which corresponds to the placement of cast-in-place concrete on bedding material or structure backfill material. A resistance factor of 0.80 should be applied to the friction factor.

4.1.3 Settlement

Since each culvert will be placed on silty sand material, immediate elastic settlement is estimated to be less than one inch. Most of the settlement is expected to occur during or shortly after construction. Differential settlements are anticipated to be less than 25% of the total (elastic) settlement.

4.1.4 Global Stability

It is not anticipated that global stability will be an issue for the slopes adjacent to the culverts due to the anticipated embankment fill sections that will be constructed adjacent to the structures. Global stability of cut and fill slopes as well as proposed wall locations is discussed in more detail in Section 4.5.

4.1.5 Corrosion

In general, the tested soils from the project sites exhibited low potential to corrode buried steel or concrete structures. No special material is required for metal pipe culverts, concrete pipe culverts, or reinforced concrete box culverts that are planned for this project.

4.1.6 Retaining Structures

Headwall and wing wall retaining structures can be supported on spread footings designed in accordance with the recommendations listed above. Retaining structures should be designed to resist lateral earth pressures and other applicable lateral loads in accordance with the AASHTO Standard Bridge Design Specifications. Lateral earth pressure is influenced by the strength of the abutment backfill, the presence of water, and the ability of the abutment or wall to move in response to lateral loads. Other loads, such as live loads, construction loads, and soil compaction loads should also be considered.

Unbalanced water behind a wall adds significant pressure and should be avoided by using structure backfill (FP-03, Section 704.04) against backfilled structures and assuring a free draining gravity outlet for captured water. Remaining backfill should consist of select granular backfill (FP-03, Section 704.10).

Where deflection of the wall can be expected, a coefficient of active earth pressure should be used for wall design. The recommended coefficient of active earth pressure is 0.30, which corresponds to an equivalent fluid pressure of 38 pcf.

4.2 SETTLEMENT REPAIRS/ MSE WALL RECOMMENDATIONS

Various types of repairs, including subexcavation and replacement with deep patch or mechanically stabilized earth (MSE) walls, were evaluated as potential solutions for repairing areas experiencing surficial distress at Sites 3 and 4. The potential for shifting the roadway alignment into the cutslope to gain the required roadway width is not considered feasible at Site 3 because of steep upslope terrain, which could result in an unstable, high cutslope or relatively expensive cut walls. Based on the analysis, a deep patch is not feasible at Site 3 or 4 due to the near-vertical slopes adjacent to the distress areas. Due to the poor condition of the two existing dry-stacked retaining walls and depth of repairs, MSE walls are recommended for the repair of settlement distress at both Sites 3 and 4.

At Site 3, the area experiencing distress is located between Station 59+10 and 60+10 LT, above a near-vertical, dry-stacked retaining wall approximately 12-feet high with a guardrail protecting the outbound traffic lane. Newton Fork Creek flows immediately adjacent to the base of the wall. The area of distress is approximately 18-feet wide and about 100 feet long with cracks extending across the centerline of the existing roadway, near the middle of the opposite traffic lane. Photographs of the distress location are included in Appendix E. Pennington County maintenance crews indicate that the top 3-feet of material has been replaced over the past 10 years. Distress is usually observed in the spring, following winter runoff events. Based on boreholes B-14A through B-17, located in the area experiencing distress, loose silty sand fill materials with varying sequences of courser rock fragments were encountered from the roadway base to an approximate depth of 10-15 feet below ground surface, where schist bedrock was encountered. The distress is most likely caused by piping of loose fill material behind the existing retaining wall.

At Site 4, the area experiencing distress is located between Station 132+25 and 132+75 LT near the outflow of Newton Pond, above a near-vertical, dry-stacked retaining wall approximately 10-feet high with a guardrail above. The area of distress is approximately 8-feet wide and about 50 feet long. The dry-stacked retaining wall elements are W-shaped with 1- to 2-feet of vertical displacement rather than horizontal, reflecting the observed surface distress. Pennington County maintenance crews indicate that the top 2 feet of asphalt has been replaced at this location over the past 10 years. Photographs of the distress location are included in Appendix E. Like Site 3, distress is generally observed in the spring, following winter runoff events. Based on borehole B-28, located in the area experiencing distress, clayey sand fill material was encountered from the roadway base to an approximate depth of 6.5 feet below the ground surface, near the base of the wall. Below this elevation, the material contained larger gravel and cobble rock fragments to a depth of about 15 feet below the ground surface, when weathered schist bedrock was encountered. Similar to Site 3 and evident from the photographs, the distress is most likely caused by piping of loose fill behind the existing retaining wall.

The MSE wall analyses follow the design methodology and guidelines in the AASHTO "Standard Specifications for Highway Bridges," 17th Edition (2002). Per AASHTO, the required minimum factors of safety for these walls are listed in Table 4-3.

Table 4-3. Required Minimum Factors of Safety

Design Component	Minimum Factor of Safety (Static)
Bearing Capacity	2.5
Overturning	2.0
Sliding	1.5
Global Stability	1.3
Eccentricity	<L/6

The MSE walls were preliminarily evaluated using the Mechanically Stabilized Earth Walls (MSEW 3.0) program developed by ADAMA Engineering. The design methodology used by MSEW is consistent with current AASHTO and FHWA guidelines for assessment of the internal and external stability of MSE walls.

The proposed MSE walls were evaluated based on the existing site conditions and available subsurface information. The engineering properties of soils used in the analyses were based on conservative estimates, as shown in Table 4-1. Cross sections were evaluated where the wall height was at its maximum and/or where the slope in front of the fill wall was the steepest. A water table was not considered in the analyses, as the wall system will include interior drainage and is considered free draining. A traffic surcharge of 250 psf was modeled in the analyses.

An MSE wall system is recommended to repair the roadway distress for Sites 3 and 4 based on the results of the subsurface investigation and analyses. It is recommended that the MSE wall system be constructed with a minimum reinforcement length equal to 70 percent of the wall height (0.7H) or 8 feet, whichever is greater. Based on the results of the analyses, the proposed MSE wall system will meet the required minimum factors of safety as described in Table 4-3. Shoring walls may be required at Site 3 for approximately 30 feet in length near the highest section of the MSE wall to ensure stability of structure excavations and allow for maintenance of traffic. A temporary lane width of 13 feet is required.

For both wall sites, the recommended minimum embedment depth is 2.0 feet or to the depth of scour potential. The use of granular soils for backfilling retaining walls is recommended to provide lower lateral earth pressures and superior drainage. It is recommended that the reinforced and retained portions of the wall system be backfilled with select wall backfill and wall backfill, respectively. Granular backfill materials should meet the requirements of Subsection 704.13 of the FP-03. Riprap protection at the base of each wall adjacent to the stream channel is recommended, in addition to the use of clean gravel backfill to the height of the 100-yr storm event to allow free drainage without settlement associated with stream water level fluctuations. Based on laboratory testing results, ***on-site, native soils will generally not meet the requirements for select wall backfill and wall backfill due to the high percent fines, high plasticity index, and low peak friction angle***. Backfill used in MSE walls must meet the physical and electrochemical requirements of Section 704 of the FP-03.

MSE wall detail drawings are provided in Appendix H and special contract requirements are provided in Appendix I.

4.3 ROCKERY RECOMMENDATIONS

Due to existing alignment constraints near Newton Fork Pond, a small cut wall will be required. For aesthetic and geometric reasons, a rockery was chosen. The rockery was evaluated using modified design methodologies based on gravity wall design (FHWA Rockery Design and Construction Guidelines, 2006), using the required minimum factors of safety in Table 4-3.

The proposed rockery was evaluated based on the existing site conditions and available subsurface information. The engineering properties of soils used in the analyses were based on conservative estimates, as shown in Table 4-1. Cross sections were evaluated where the wall height was at its maximum, approximately 10 feet. A water table was not considered in the analyses, as the wall system will include interior drainage. Seismic loading was neglected in external and global stability analyses.

Based on the results of the analyses, the proposed rockery wall system will meet the required minimum factors of safety as described in Table 4-3, with a minimum base width of 6 feet. The base rock should also be embedded a minimum of 1 foot.

The rockery should be backfilled with free-draining, minus 6 inch, granular materials. This granular material will also serve as the drainage system for the wall and reduce the buildup of pore water pressures behind the wall. A 4-inch-diameter perforated PVC collector pipe should be installed within the drainage layer near the base of the wall. Type IV-E geotextile should be placed along the backslope prior to placement of the drainage layer. Detail drawings of the proposed rockery wall are provided in Appendix H and special contract requirements are provided in Appendix I.

4.4 ROCK CUT ANALSYSIS AND RECOMMENDATIONS

There are three potential rock cut areas on the Hill City to Lead project, located at Sites 1, 2, and 3. The following sections discuss the surface observations, geometry of the discontinuities, kinematic analysis, and recommendations for slope angles at each rock cut.

4.4.1 Site 1 Rock Cuts– Station 41+00 to 48+00

At Site 1, two rock cut areas are planned. Rock cut 1A, from Station 41+00 to 42+00 RT, is planned to cut approximately 10-15 feet into the existing slope at a maximum cut slope angle of 1V:1H, resulting in a maximum cut height of approximately 40 feet. The foliation in the schist is angled nearly perpendicular to the roadway alignment. Rock cut 1B, from Station 42+80 to 48+00 RT, is planned to straighten the existing curve, cutting back about 45 feet at a maximum cut slope of 1V:1H, resulting in a maximum cut slope height of approximately 30 feet. The foliation of the schist is nearly parallel to the roadway at the middle of the curve as seen on existing rock cuts, so this plane of weakness will be the controlling factor in the final cut slope orientation. The existing rock cut angles at Site 1 were controlled by the foliation of the schist and have performed well according to the local roadway maintenance crew. Schist at this location is harder at the surface than the other outcrops on Site 1. There are no current maintenance issues associated with any rock cuts on Site 1 and the ditches require cleaning very infrequently. No impact marks were visible in the existing traffic lanes, indicating that the existing geometry is relatively stable. In the ditch, the maximum rock size observed was a flat sheet approximately 1-foot wide by 1-foot high by 3-inches thick, while most particles were broken down to large gravel and cobble-sized plates. The plate-like structure indicates that the weakest fracture plane is along the foliation of the schist. A panoramic photo, plan view, and cross-section of the rock cuts at Site 1 are included in Appendix G.

The foliation of the schist strikes parallel to the roadway and is steeply dipping towards the roadway, resulting in exposed planar faces. This plane of weakness is the most likely plane for sliding. Two joint sets were identified based on surface observations that are generally discontinuous and variable within about 10-degrees. A kinematic analysis was conducted to determine the possibility of sliding or wedge failure along the foliation and joint sets. Plots of discontinuities were completed using the DIPS software, version 5.1, a RocScience (2003) plotting program. For the kinematic analysis, the orientation of the roadway in relation to the rock cut and an assumed friction angle of the schist of 23-degrees were used. The result of the analysis, using the Markland test for wedge failure analysis (Hoek and Bray, 1994), indicates that there is a geometric possibility of wedge and sliding movement at this rock cut. Sliding is possible along the foliation of the schist with cross-joint release at 90-degrees. Wedge sliding is possible along the intersection of Joint Set 3 and the foliation plane. Additionally, toppling is also geometrically possible due to the orientation of Joint Set 1 with respect to the rock cut and the joint set's near-vertical dip. Neither wedge sliding nor toppling were observed in the existing rock cuts. Pole plots of the discontinuity sets and the results of the analysis are included in Appendix G.

In summary, there is low rockfall risk anticipated at this location because the planned rock cut orientations are similar to the existing rock rocks, where the maintenance history indicates there is good ditch catchment coupled with small-sized rock fragments eroding from the outcrops. It is recommended that the existing slope angle of 1V:1H is used for the design of the rock cut and a minimum ditch with of 10 feet is maintained. Access to the top of the rock outcrop is possible, which could eliminate the need for special construction equipment. The county maintenance crew has been able to excavate similar weathered schist without blasting, but it will be up to the Contractor to evaluate the on-site conditions prior to construction to determine if blasting is necessary. There are no signs that the existing rock cut was blasted. The top of the slope should be rounded in to the hillside for aesthetics and additional safety.

4.4.2 Site 2 Rock Cut – Station 105+70 to Station 108+50

A large rock cut exists from Station 105+70 to 108+50 at Site 2, that will be cut back about 15 feet, which will result in an approximately 50 feet high cut. The existing cut slope is about 40 feet high at a slope of 2V:1H with an approximately 10-foot ditch width. There are no current maintenance issues associated with this rock cut and the ditches require cleaning very infrequently. No impact marks were visible in the existing traffic lanes, indicating that the existing geometry is relatively stable. In the ditch, the maximum rock size observed was a flat sheet approximately 3-feet wide by 1-foot high by 3-inches thick, while most particles were broken down to large gravel and cobble-sized plates. The plate-like structure indicates that the weakest fracture plane is along the foliation of the schist. A panoramic photo, plan view, and cross-section of the rock cut at Site 2 are included in Appendix G.

The foliation of the schist strikes parallel to the roadway and is steeply dipping towards the roadway, resulting in exposed planar faces. This plane of weakness is the most likely plane for sliding. Three joint sets were identified based on surface observations that are generally discontinuous and variable within about 10-degrees. A kinematic analysis was conducted to determine the possibility of sliding or wedge failure along the foliation and joint sets. Plots of discontinuities were completed using the DIPS software, version 5.1, a RocScience (2003) plotting program. For the kinematic analysis, the orientation of the roadway in relation to the rock cut and an assumed friction angle of the schist of 23-degrees were used. The result of the analysis, using the Markland test for wedge failure analysis (Hoek and Bray, 1994), indicates that there is a geometric possibility of wedge failure along the intersection of Joint Set 1 and the foliation plane of the schist and along the intersection of Joint Set 3 and the foliation plane of the schist at this rock cut. Sliding is possible along the foliation of the schist with cross-joint release at 90-degrees. Toppling is also geometrically unlikely. Pole plots of the discontinuity sets and the results of the analysis are included in Appendix G.

In summary, there is low rockfall risk anticipated at this location due to the maintenance history indicating good ditch catchment coupled with small-sized rock fragments eroding from the outcrop. It is recommended that the existing slope angle of 2V:1H is used for the design of the rock cut and a minimum ditch with of 12 feet. Access to the top of the rock outcrop may be possible, which could eliminate the need for special construction equipment. The county maintenance crew has been able to excavate similar weathered schist without blasting, but it will be up to the Contractor to evaluate the on-site conditions prior to construction to determine if blasting is necessary. There are no signs that the existing rock cut was blasted. The top of the slope should be rounded in to the hillside for aesthetics and additional safety.

4.4.3 Site 3 Rock Cut – Station 56+80 to Station 61+80

At Site 3, a rock cut exists from Station 56+80 to Station 61+80 that is about 10-15 feet high adjacent to the roadway, then a steep slope projects up the hillside another 500 feet at a slope of about 1V:1H, intermittently intersecting more bedrock. The existing ditch is too narrow and does not always collect slope debris before it falls into the roadway. Material falling into the roadway is generally gravel-sized and quickly erodes to sand-size particles. Because of this observation, the currently a 2-foot wide unpaved ditch will be replaced with a 4-foot paved ditch at the base of the cut slope. Instead of cutting into the steep topography at this location, it is recommended that an MSE wall be constructed on the outward fill slope to widen roadway and repair distress above the existing dry-stacked retaining wall. The plate-like structure indicates that the weakest fracture plane is along the foliation of the schist. A panoramic photo, plan view, and cross-section of the rock cut at Site 3 are included in Appendix G.

The foliation of the schist strikes parallel to the roadway and is steeply dipping towards the roadway, resulting in exposed planar faces. This plane of weakness is the most likely plane for sliding. Three joint sets were identified based on surface observations that are generally discontinuous and variable within about 10-degrees. A kinematic analysis was conducted to determine the possibility of sliding or wedge failure along the foliation and joint sets. Plots of discontinuities were completed using the DIPS software, version 5.1, a RocScience (2003) plotting program. For the kinematic analysis, the orientation of the roadway in relation to the rock cut and an assumed friction angle of the schist of 23-degrees were used. The result of the analysis, using the Markland test for wedge failure analysis (Hoek and Bray, 1994), indicates that there is minimal geometric possibility of wedge failure at this outcrop. However, sliding is possible along the foliation of the schist with cross-joint release at 90-degrees. Toppling is also geometrically unlikely. Pole plots of the discontinuity sets and the results of the analysis are included in Appendix G.

In summary, avoiding this rock cut area and adding paved ditch width to catch loose debris is recommended. Instead of cutting into the steep topography at this location, it is recommended that an MSE wall be constructed on the outward fill slope to widen roadway and repair distress above the existing dry-stacked retaining wall.

4.5 SLOPE RECOMMENDATIONS

The recommended slope ratios are based on site observations of the existing cut and fill slopes along the route that appear to be stable. Generally, the slopes of the fill embankment along the roadway vary between 1V:4H to 1V:2H and the cut slopes in soils vary between 1V:2H and 1V:1.5H. For cut slopes in silty sand material, maximum slopes of 1V:1.5H are recommended. For fill slopes, maximum slopes of 1V:2H are recommended. Reinforced soil slopes or retaining walls may need to be designed if steeper slopes are required. Both of these cases would need further analysis and additional recommendations developed for site specific conditions. Cut slopes in rock are recommended to mirror the existing rock cut slope angles as discussed in Section 4.4.

Global slope stability analysis was performed on proposed cut and fill slopes and walls at each site using Slide, Version 5.0, the two dimensional, limit equilibrium computer program from RocScience. The Simplified Bishop method of slices was used with isotropic soil and rock parameters for the slope stability analysis of static conditions. The engineering properties of soils used in the analyses were based on conservative estimates, as shown in Table 4-1. Cross sections were evaluated where the wall or slope height was at its maximum and/or where the slope in front of the fill wall was the steepest. A water table was not considered in the analyses, as the wall system will include interior drainage and is considered free draining. Additionally, the on-site materials are generally well draining. A traffic surcharge of 250 psf was modeled in the analyses. A minimum factor of safety of 1.3 was used for evaluation of static conditions.

Table 4-4 provides a summary of the preliminary global stability analyses for the proposed fill and cut slopes and walls. Global stability analysis for analyzed cross sections is provided in Appendix F.

Table 4-4. Summary of Global Stability Analysis

Site	Station	Cut/Fill	Slope/ Wall	Total Wall or Slope Height (ft)	Slope Angle (Degrees)	Factor of Safety (Static)
1	41+50	Rock Cut	Slope	38 feet	1V:1H	2.09
2	106+50	Rock Cut	Slope	50 feet	2V:1H	1.30
3	60+50	MSE Fill	Wall	17 feet	Near Vertical	1.82
4	132+50	Rockery Cut	Slope	9.5 feet	4V:1H	1.64
4	132+50	MSE Fill	Wall	8 feet	Near Vertical	2.60

4.6 SHRINK/SWELL RECOMMENDATIONS

On-site soils encountered along most of the alignment generally consist of alluvial silty sands with some gravel. It is estimated that such soils will have a shrink percentage of 11 percent, corresponding to a shrink/swell factor of 0.90.

The majority of the material excavated for Sites 1 and 2 will be in weathered schist bedrock material in order to balance the earthwork along the project. It is estimated that the excavated, weathered rock will have a swell percentage of 12 percent, corresponding to a shrink/swell factor of 1.12. Table 4-5 summarizes station ranges for anticipated shrink/swell factors for the project cut slopes.

The recommended slope ratios are based on site observations of the existing cut and fill slopes along the route that appear to be stable. The recommended shrink/swell factors are based on a combination of standard tabled values for common materials in the FLH Technical Guidance Manual (2006) and experience with other CFLHD projects in similar materials.

Table 4-5. Summary of Shrink/Swell Locations

Site	From Station	To Station	Offset	Anticipated Material	Recommended Shrink/Swell Factor
1	31+00	41+00	LT & RT	Silty Sand	0.90
1	41+00	42+00	RT	Weathered Schist	1.12
1	42+00	42+80	LT & RT	Silty Sand	0.90
1	42+80	48+00	RT	Weathered Schist	1.12
1	48+00	49+00	RT	Silty Sand	0.90
3	49+00	56+80	RT	Silty Sand	0.90
3	56+80	61+80	RT	Weathered Schist	1.12
3	61+80	100+00	RT	Silty Sand	0.90
2	100+00	105+70	RT	Silty Sand	0.90
2	105+70	108+50	RT	Weathered Schist	1.12
2	108+50	120+50	RT&LT	Silty Sand	0.90
4	120+50	137+00	RT&LT	Silty Sand	0.90

4.7 CONSTRUCTION CONSIDERATIONS

The following sections discuss site specific issues that should be considered during the development of the project specifications. Miscellaneous details and drawings that are appropriate for this project are included in Appendix H and Special Contract Requirements (SCR) are included in Appendix I.

4.7.1 Site Preparation

Clearing and grubbing of the project sites should be performed in accordance with Section 201 of the FP-03. In general, it is not anticipated that any areas of difficulty will be encountered during the clearing and grubbing operation.

Based on conditions encountered during the subsurface investigation, it is not likely that a significant depth of topsoil will be present at the project sites. For estimation purposes, it should be assumed that limited topsoil will be stripped and stockpiled for re-use on the project. Topsoil is anticipated to be imported for the project.

Silty sand is the predominant soil type found along the project, and was generally classified as A-2-4, or A-4 soils with low plasticity. These materials exhibit good characteristics for use as embankment material. The A-4 designation of embankment soils will require either headwalls at the corrugated metal pipe inlets or an impermeable clay seal around the inlet to prevent piping. Occasionally, there are areas of sandy clay that classified as A-6 material, which may not meet all the requirements for embankment material. The volume of material anticipated to be wasted is negligible. Additionally, laboratory test results indicate a majority of the fines percent consist of silts, which can be difficult to compact and may be susceptible to frost heave differential movement. Native soils used in embankments should be compacted to 95% of maximum density according to the requirements of Section 204 of the FP-03. Due to the silt content, it may be necessary to compact dry of the optimum moisture content to achieve 95% compaction. Five-point proctor testing is recommended to verify the behavior of the on-site material prior to compactive efforts. Good roadway drainage is a key element to preventing frost heave.

4.7.2 Grading Requirements

Due to the limited amount of local borrow sources or waste areas, and to maximize cut to fill balance on this project, it is anticipated that the majority of roadway excavation will be used in the construction of roadway embankments. It is anticipated that a majority of native materials will meet the requirements of embankment fill and bedding material, per Section 704 of the FP-03.

The foundation for embankment construction not associated with the box culvert foundation excavation can be prepared according to FP-03 Section 204.09(a). For separation between the existing ground and bedding material or structure backfill material, a **Type IV-E geotextile** (FP-03, Table 714-2) should be used, based on the assumption of native, silty sand material. This geotextile is designed for the native soils with regard to retention, clogging, and permittivity. The majority of the on-site soils have a fines content between 15 and 50 percent. This geotextile selection corresponds with the CFLHD Hydraulics recommendations for use under riprap applications, which simplifies the contract documents by requiring only one type of geotextile for the project.

Embankments will be raised at the Mickelson Trail crossings at Site 1 and Site 2 to accommodate the installation of precast culvert underpasses. At Site 1, the embankment will be constructed on the roadway fill material or alluvial silty sands. At Site 2, the embankment will be constructed partially on weathered schist bedrock and partially on alluvial silty sands. Settlement issues with the embankment construction are not anticipated, but constructing a large fill should be done with careful compaction control. It is especially important to compact the side slopes of the embankment, either by overfilling and cutting back to the design slope, or hand-working compaction at the face.

Special underdrain construction is designed at Site 2. Special 605-A (Appendix H) was developed to address a natural spring that outlets near Station 112+50 that will be covered by the new roadway embankment that is being raised to accommodate the new Mickelson Trail underpass. Key elements include perforated collector pipes, a geocomposite sheet drain at the spring outlet location (near the existing headwall), and an outlet pipe that drains to Newton Fork Creek.

It is anticipated that roadway excavations conducted along sections of Site 1, 2, and 3 may encounter rock requiring carefully planned and uniquely adapted blasting approaches to achieve engineered road cuts that are structurally sound and aesthetically pleasing. Controlled blasting methods that utilize the natural geologic bedding planes and joint structured in a predicted and controlled manner to form the final cut slopes and to minimize back break beyond the trim line should be used. The contractor will be required to make the final determination on the rippability characteristics of encountered material based on review of the boring logs and equipment capabilities. Pre-blasting and post-blast surveys will be required for structures close to blasting activities. A blasting SCR is included in Appendix I.

Limited slope scaling at Site 3 is recommended due to loose blocks of schist observed on the existing rock cutslope. Earthwork equipment with a 20-foot reach is expected to reach the loose debris, with no hand scaling required. This should be paid for as part of the roadway excavation. For clear communication with the contractor, a pre-construction meeting could be held to discuss the scope and location of rock scaling activities. A slope scaling SCR is included in Appendix I.

5.1 LIMITATIONS

The recommendations in this report are based on the data obtained from exploratory borings, field review, and the laboratory test results. The results of these explorations and tests represent conditions at the specific locations indicated. Subsurface variations across the site are likely and may not become evident until excavation is performed. The Analysis and Recommendations sections in this report include interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgment of the Contractor.

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